Strength-Deformation Behavior of Saturated Clays

and Drained/Undrained Stability (Parts D&E of Outline)

I. STABILITY PROBLEMS AND DRAINED STRENGTH PARAMETERS

A. Classes of Stability Problems & Types of Stability Analysis

Review of 1.361 Part IV-3

B. Determination of Effective Stress Failure Envelopes for CD Case

1. Use of CD & CU Tests
   1) CD OS 2) CD TX 3) CU TX

2. Miscellaneous
   1) Variations in ESE: OCR = 1
   2) " " " : High OCR
   3) Common triaxial testing problems
   4) Comparison of ESE and correlation

C. Long Term (CD Case) Stability: Problem Soils

C.1 Stiff Fissured and Stratified Clays & Clay Shales

1) Introduction
   2) Definition of 3 envelopes (peak, fully softened & residual)
   3) Measurement of residual envelope
   4) Overview of fully softened vs residual envelope

5, 6, 7) Recommendations for selecting c' & p' as per 1995
   and results from recent research

8) Basic Research on p'
   9) Empirical correlation

C.2 Highly Structured, Sensitive Clays (Quick Clays)

1) Background
   2) Nancy
   3) parce

C.3 Conn. Valley Varved Clays

Handout Sheets

IA/2

IB

IC
**Mini-Problem No. 1 on Strength of Clays**

<table>
<thead>
<tr>
<th>Topic in HW Notes</th>
<th>Approx Date</th>
<th>Questions</th>
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<tr>
<td>I B Measurement of C' &amp; φ'</td>
<td>4/4/01</td>
<td>1) What can cause major errors in the measurement of C' &amp; φ' for CD analysis of homogenous cohesive soils?</td>
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<tr>
<td>I C Problem Solns</td>
<td>4/5/01</td>
<td>1) For cuts in cliff, assume 3 sheeted soils. a) When safe to use peak envelope? b) When NC? c) When must use φ' or? d) When get combination of above? 2) What is effect of using undrained vs remolded clays on value of φ' 3) For cuts in natural slopes in quick clays, is CD safe? If yes, is peak CD safe?</td>
</tr>
<tr>
<td>II A Suff UWC use</td>
<td>4/9/01</td>
<td>1) FVT a) Why decrease decreases with increasing AE? b) What makes a unsafe to use AE? 2) CPTU a) How is Nk determined? b) What is the major cause of problem in getting consistent φ profiles in soft clays? 3) DMW: How reliable is $\frac{S_u}{Q_o} = 0.22(O_{CR})^{0.8}$ when $O_{CR} = (0.5kg)^{1.55}$? 4) When would you replace curing compaction test with a UWC test ($T_c &gt; 0.5o$)? 5) Ald Bishop &amp; Ogilive (1960) conclude that both UWC &amp; $S_u$ (CD) reliable $S_u$ for UWC?</td>
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# Class Schedule & Reading Assignments: Stability & Strength of Cohesive Soils

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<tr>
<th>Topics</th>
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<th>'85 SQA SF</th>
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<td>4) Sample</td>
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<td>6) Time Effects:</td>
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<td>7) CU Case</td>
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<td>8) Home Problems</td>
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**NOTE:** There will be a series of mini-problems, mostly in the form of questions for class discussion on Topics 1-7. Topic 7 will have a major design problem.
Strength-Deformation Behavior of Saturated Clays
And Drained/Undrained Stability (Part 1 of Outline)

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   2) " High OCR
   3) Common triaxial testing problems
   4) Comparison of ESE and correlations

C. Long Term (CD Case) Stability: Problem Soils

4/3/01  CHARLES C. LADD

Wed 4/4/01 since
reviseing updating not
yet finished
IA: Classes of Stability Problems and Types of Stability Analyses (Sheet A2; Table 6)

CASE 1: UU Case (Undrained)

- Embankment on Soft Clay

Also BC

Also BC

TSA = Total Stress Analysis

Note: Also can/should use USA

CASE 2: CD Case (Fully Drained)

- Slow constr or long time → $U_e = 0$
- Slow failure → $U_s = 0$

Next: Testing → $c' + \phi'$

CASE 3: CU Case (Partial drainage prior to Undrained Failure)

- $U_e > 0$
- $U_s > 0$

CCL Methodology: Use CKw

QRS: "" UU/CU (Sheet A2; Fig. 3)

Critical Conditions (as for 1.36)

Loading (Construction → $+\Delta P$)

- Footings, tanks, emb, dem, ...
- UU critical since $U_e$
  → drainage → incr. str.
  (esp. low OCR)

Unloading (Construction → $-\Delta P$)

- Excavations, etc.
- CD critical since $-U_e$
  → drainage → decr. str.
  (esp. high OCR with $-U_s$)
### TABLE 6. Stability Problems Classified According to Drainage Conditions and Definition of Factor of Safety

<table>
<thead>
<tr>
<th>Case</th>
<th>Common description</th>
<th>Proposed description</th>
<th>Proposed classification</th>
<th>Definition of factor of safety&lt;sup&gt;a&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Undrained, short-term or end-of-construction</td>
<td>No consolidation of soil with respect to applied stresses and undrained failure</td>
<td>Unconsolidated-undrained = UU case</td>
<td>(rac{q_f}{\sigma'} = \frac{\sin \phi_{cu}}{1 - \sin \phi_{cu}}) (Terzaghi &amp; Peck 1967)</td>
</tr>
<tr>
<td>2</td>
<td>Drained or long-term</td>
<td>Full consolidation of soil with respect to applied stresses and drained failure ((u = 0))</td>
<td>Consolidated-drained = CD case</td>
<td>(c_{u} = c_{f} \tan \phi_{cu})</td>
</tr>
<tr>
<td>3</td>
<td>Partially drained or intermediate</td>
<td>Partial or full consolidation of soil with respect to applied stresses and undrained failure</td>
<td>Consolidated-undrained = CU case</td>
<td>(c_{u} = \frac{c_{f}}{\tan \phi_{cu}}) (Eq. 8)</td>
</tr>
</tbody>
</table>

<sup>a</sup>\(r_{m}\) = mobilized shear stress required for equilibrium; \(s_{z}\) = undrained shear strength obtained from conventional testing associated with typical \(\phi = 0\) analyses; \(c_{u}\) = undrained shear strength obtained from techniques recommended in Section 5; and \(s_{z}\) = drained shear strength defined in Eq. 1.

---

**FIG. 2. Angle of Shearing Resistance \(\phi_u\) from Isotropically Consolidated-Undrained Triaxial Compression (CIUC) Tests as Defined by A. Casagrande**

**FIG. 1. Conventional Effective Stress Analysis Applied to Critical CD Case for Unloading Problem**

**FIG. 3. Comparison of Effective Stress and Undrained Strength Analyses for Evaluating Stability during Staged Construction**
IE DETERMINATION OF EFFECTIVE STRESS FAILURE ENVELOPE
FOR CO CASE (Saturated Natural Cohesive Soils)

1. USE OF CD:CU TESTS

   Note: Limited data on “ordinary” clay indicates that if has
   little effect on values of $c'$ and $\phi'$ (viz., using $t_f > 100$ required obtain $c_s = \phi$)

   1.1 CD Direct (Box) Shear Tests

   a) Advantages
   - Simple equipment & easy to run
   - Low cost
   - Short $t_f < 1$ day

   b) Disadvantages
   - Non-uniform shear strain condition $\rightarrow$ no $\tau_n$ & $\delta$ data

   2) Unknown shear condition at failure

   a) $\tau_n = \tau_f; \delta = 45^\circ; \sigma_n/\sigma_v = \tan \phi'$
   b) $\tau_n = \pi_0; \delta = 45^\circ; \sigma_n/\sigma_v = \sin \phi'$

   $\tau_n/\sigma_v = 0.5 \rightarrow \phi' = 26^\circ \text{ (a)}$
   $= 30^\circ \text{ (b) \hspace{1cm} std. practice}$

   3) Tilting at high $\tau_n/\sigma_v$
   - Tensile $\sigma$ at leading portion
   - Compressive $\sigma$ at trailing portion

   4) Run test too fast ($t_f < 10\text{ time}$)

   - Drained, $u_s = 0$
   - Too fast, $v_s > 0$
   - Hand Cranked (HAND CRANKED)
   - Measured $c'$ too high $\rightarrow$
   - Unsafe $FS$ for shallow slope failure
1.2 CD Triaxial (Usually CIQC + U since Kc does not affect c', $\phi'$)

a) Advantages

i. Known soil conditions and meaningful stress-strain data
ii. Can vary ESP to define ESE at low stress
iii. Most reliable

b) Disadvantages

i. More complex equipment & harder to run
ii. Much longer time (1-2 weeks) & more expensive

1.3 CU Triaxial (Usually CIUC)

a) Advantages

i. Obtain information on undrained behavior for CU Case
ii. Less time than CIQC

b) Disadvantages

i. Procedures more complex to ensure reliable ESE data
ii. Cannot define ESE at low $\phi'$ for high OC soils

3) Varying ESE at $q_c$, max obs $\phi$ tangency (See 2.1.3.2)

Note: Generally use max obs. or tangency to estimate $c', \phi'$ for CD Case

4) Potential large unsafe errors in $c'$ if do not attain pre-pressure equalization (See 2.3)
2. MISCELLANEOUS

2.1 Variation in ESE: OCR=1 (CUC Tests)

1. Simple clay type behavior: $q'_f = q'_o$
2. Sensitive clay: $q'_f < q'_o$
3. Archie: not really NC Cohesiveless (small +4)

- Value of $q = f(mobilized \dot{q})$ x (magnet of $p'$)
- High S, $q'_f < q'_o$ by 5-10%
- $\text{CUC } \phi'_o(q_b = q_o) = \text{CUC } \phi'_o - 10 - 30$

2.2 Variation in ESE: High OCR (CUC Tests)

- Usually select ESE at Max. Obl. or tangency to estimate ESE for CUC Test
- Bul extrapolated envelope is too high at low $p'$ (See 1Bd)

2.3 Common Triaxial Testing Problems (Germes/Ladd, 1983 ASCE 57-97b)

a) Piston Friction (CU/CD)

- Need bell bearing - nothing diaphragm or internal load cell for reliable $q_f$ - $q_b$
- Solid bushing needed - semi-infinite
b) Filter Strips (CUSCO)
   - Compression: 10 cm² x 8 cm. Typical correction Δg = 100 psi = 5 kPa
   - Extension: Need spiral + pull at notches → maximum flexibility

c) Area Correction (CUSCO)
   - Compression: See G11(88) for cylinder, parabolic + bulging
   - Extension: Discount data when notches necking occur


d) Saturation (CU)
   - Need min. Ψ_b = 2-3 atm. Always check that B ≥ 95 % few minutes

e) Fractured Ends - Pre-pressure Equivalent at "High" OCP (CIVC)

\[
\begin{align*}
\sigma_1 - \sigma_3 & \quad \text{CIVC at "Fast" Rate} \\
\Delta u & \quad \text{Measured } \Delta u \\
\Delta u & \quad \text{Actual } \Delta u \\
\end{align*}
\]

\[
\begin{align*}
\Delta u & = \Delta \sigma_a \\
H_2O & = \text{migration} \\
\end{align*}
\]

\[
\begin{align*}
\text{Measured ESE} & \rightarrow \text{Actual ESP} \\
\text{Actual ESE} & \rightarrow \text{Slow stirring} \rightarrow \text{no decrease pre-pressure} \\
\end{align*}
\]

Correct ESE Mismatch: 1) Either Ψ measured at fast (correct Ψ) (with fractured ends)
                      2) Or very slow if Ψ measured at base (Ψ too low)
2.4 Comparison of ESE and Correlations

a) Natural BBC: CK, UC/E; OCR = 1.5 – 6; Max. Obl.: (CANT Project)

<table>
<thead>
<tr>
<th></th>
<th>$c'/\sigma_{vm}$</th>
<th>$\phi'$</th>
<th>$c'/\sigma'$</th>
<th>$\phi'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>TC</td>
<td>0.017</td>
<td>28.5</td>
<td>0.044</td>
<td>29</td>
</tr>
<tr>
<td>TE</td>
<td>0.055</td>
<td>19</td>
<td>0.031</td>
<td>29</td>
</tr>
</tbody>
</table>

Large difference

TC vs. TE

Similar values

TC vs. TE

NOTE: We will discuss different between TC vs. TE more fully under ICC

b) Friction Angle vs. Plasticity Index: Normally Consolidated Soils

1) NAVDOCKS DM-7 (1961)


FIG. 2. Values of Friction Angle $\phi'$ for Natural Clay Compositions
Relationship between Cohesion Intercep and Preconsolidation Pressure

$\sigma'_n$ in $\sigma'_n/\sigma'_p$ Range of
- 2 to 5
- 10 to 20

$C'/\sigma'_p = 0.10$
$C'/\sigma'_p = 0.05$
$C'/\sigma'_p = 0.01$
$C'/\sigma'_p = 0.024$

Excludes cemented Champlain clays

Note: Fig. 8 of paper shows $m = 0.3 - 0.9 \times \sigma'_n$ in $\sigma'_p$, 20-80%
LONG TERM (CO) STABILITY: PROBLEM SOILS

C1 STIFF FISSURED & STRATIFIED CLAYS AND CLAY SHALES

1.1 Introduction

1) Critical Condition

\[ \sigma' = \tau_f = C' + (\sigma - u_s) \tan \phi' \]

\[ F_S = \frac{\sigma'}{\tau_m} = \frac{\tan \phi'}{\tan \phi_m} \]

2) Values of \( C' \) & \( \phi' \) to use in analysis depend on:

a) 1st time vs. pre (activated) slide.

b) Homogeneous (also partly for PI & CF value fraction) vs. Non-homogeneous (NH) (stiff clay, clay shales that contain:

1) Fissures = small, random oriented discontinuities (like closed cracks; some may be slickeneded = "polished")

2) Bedding planes-laminations: These are especially important if more plastic than "bulk" soil and have a higher initial degree of parallel particle orientation.

NOTE: NH can have either or both (plus other features such as joints and faults, although these more typically associated with rock.)

3) To appreciate problem with selection of \( C' \) & \( \phi' \), need to understand differences in ESE as function of degree of shearing.

---

No. 5505

Engineer's Computation Pad
1.2 Definition of 3 Envelopes (e.g. Skempton 1964, Gazst. 140(1), 77-100)

Results from CDOS tests on stiff, fissured London Clay (LL = 80, PI = 50, CC = 55)

1) Peak Envelope ($C'_p$, $\phi'_p$)
   - Magnitude = function of age of specimen if fissured
   - Parallel to unconfined compression
   - Example for stiff, London Clay (Skempton & Hutcham 1969, ICPIFE)
     1. Intact $C'_p$ = 1500 psi $\phi'_p$ = 28°
     2. Along fracture $\sigma'$ = 140 $\sigma'$ = 18.5°

2) Residual ($\phi'_r$) [Note: Will later see that actually curved]
   - Shear to very large displacements leading to maximum rotation of particle contacts
   - If CF > 50%, high PI, get smooth "polished" surface

3) Fully Softened ($\phi'_s$) [Note: also may be curved]
   - Corresponds to "critical state", i.e. peak strength
     when shearing OCR = 1 specimen = steady state strength of OC specimen
   - Skempton also postulated that there "minor shear not yet linked into a continuous surface" at fully softened state
1.3 Measurement of Residual Envelope

1) Material Property
- Same value whether hot
  undisturbed or remolded or
  NC or OC
- Little effect of strain rate (incluying test)
- But must shear sufficiently to attain
  more particle orientations
- Plot $t_\text{int}/t_\text{nor}$ vs $\log S$ (displacement)

2) Testing Methods
   a) Repeated direct shear, i.e. shear
      $\delta_i$, push back, shear again, etc.
      - Either pre-cut natural (to greatly reduce $S$)
      - Or remove consolidate on plate (not used)
   b) Rotational shear = ring shear

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<th>OD</th>
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<td>7.1</td>
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<tr>
<td>15.2</td>
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<td>1.9</td>
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<tr>
<td>10</td>
<td>7</td>
<td>0.5</td>
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</table>

- Typical $\delta_i$ / t = 0.1 cm/day

3) Some Results
- See Fig. 14 below - same results in pre-cut vs intact (needed much larger $S$)
- Also note reducing $t_\text{int}/t_\text{nor}$ with increasing $t_\text{nor}$ = curved envelope

Relationship between normalized shear stress and effective normal stress for
Portuguese Bend bentonitic tuff. Ca Mont. LL = 98 PI = 61 CF = 68

Figure by MIT OCW. Adapted from: *Stark & Eid (1993)* GTJ, A770, 16(1)
1.4 Overview of Softened (Critical State) and Residual \( \phi' \)

Lupini et al. [1981, Jot 13(1)] Skempton [1985, Jot 35(1)]

- A Granular particles in contact (clay fill voids)
  - \( \phi' = \phi_c \) [rolling shear]

- B Granular particles "floating" in clay matrix \( \phi' \)
  - \( \phi' \) due sliding shear
  - Controlled by clay matrix
  - \( \phi' \) much higher

- C Transitional zone with increasing difference between \( \phi_c \) and \( \phi' \)

1.5 Recommended Selection of \( c' \) and \( \phi' \) Until Aprx Mid-1990s

(Mostly by Skempton, e.g. 1970 [Jot 20(5)]; 1977 [9th IC SMFE Vol 3]

1) Along PRIOR FAILURE SURFACE having remnants of \( s \leq 1-2 \text{m} \)

Must use \( \phi' \) independent of age of prior failure

- CO-DS tests on block sample from 10-yr old failure at Mangla Dam \( = \tau_{xx} \cos \phi' \tan \phi' \)

2) 1st time failure, HOMOGENEOUS CLAY (no fissures or stratification, etc.)

Can use \( c' = \phi' \) (CCL NOTE: Only if shearing does not = shear softening after peak, which is not likely for OC clay)

3) 1st time failure, FISSURED CLAYS

- For London clay, use \( c' = 0 \) \( \phi' = \phi_c \); i.e. softening of fissures
  - Due to splitting and localized shearing = fully softened condition
  - Empirical observation from back analysis of case histories

- For some fissured clays, may get \( \phi' < \phi_c \)

E.g. Stokk & Sia [1977, JGGE 123(4)] - Analysis of 14 failures involving

- Half fissured clay \( \tau_m = \frac{1}{2} (\tau_s + \tau_f) \) for \( \tau_f > 60 \text{kPa} \), etc.
  - Half way between fully softened & residual)
1.6 Results of Research by Potts et al. (1997) "Delayed collapse of cut slopes in stiff clay" (J. GBD 77(5), 953–982.

1) Conducted coupled FE analyses (i.e., included b) of cut slopes in a stiff softening clay (patterned after London clay).

Note: Also did analyses without strain softening \( \Delta \tau \) and higher

\( \bar{\tau}_a = \text{average } \tau_a \) on rupture surface. Inc. suction also much longer \( f_p \)

(As. vegetation on slope helps).

2) Results for \( H=10 \text{ m}, \; 1:4:3 \)H as \( f(K_0) \): Collapse=failure when analysis showed abrupt increase in \( \phi_h \) at mid-slope.

These results show that

\% of slope at residual (R), at peak (P), intake (B) and not even at failure (NF) varies as \( f(K_0) \).

Varying slope height and angle also \( \rightarrow \) varying percentage

\[ R = \frac{T_p - T_f}{T_p - T_r} \]

Fig. 21. Rupture surfaces predicted by the analyses on 3:1 slopes, 10 m high, with surface suction 10 kPa and varying \( K_o \).
3) Results showing strain contours for H=10 m, W=3 H and Ko=0.5 at t=9 yr. at t=14.5 yr.

- **Note:** Accumulated deviatoric plastic strains during excavation $e_o^P < 5\%$.

- **Note:** Strain-softening starts when $e_o^P = 5\%$. It is complete when $e_o^P = 20\%$.

![Diagram showing strain contours](image)

9 years After Excavation

- $t = 9 \text{yr}$
  - $e_o^P = 20\%$
  - $e_o^P = 5\%$
  - Rupture Surface

14.5 years After Excavation - Just Before Collapse

- $t = 14.5 \text{yr}$
  - $e_o^P = 50\%$
  - $e_o^P = 20\%$
  - Residual
  - Peak
  - Rupture Surface
  - Between

**Typical Analysis (S3):** 3:1 slope, 10 m high, $K_o = 1.5$, Surface Suction 10 kPa. Contours of Accumulated Deviatoric Plastic Strain, $e_o^P$

Figure by MIT OCW.

**Conclusions**

- Odeared shear of all OC clay on slope will undergo strain-softening
- Increased degree of strain-softening from peak to fully softened (NC) to residual will decrease $t_s$ (less pre-pressure dissipation and swelling failure)
- Get progressive failure mechanism starting at toe and moving upslope
- No simple envelope at failure
1.7 Mesri et al. (SQA paper submitted to JGGE, 3/01)

1) Analysis of 100 case histories of failures (1st time & reactivated)

2) Principal conclusions

a) Most stiff clays & clay shales are NOT homogeneous. Rather, usually stratified (bedding, laminites, etc.) and insufficient
b) Stratified layers often more plastic and weaken and require less displacement to reach residual condition; may be at residual before excavation or due to undrained shear during excavation.

c) In many cases, stratification often leads to formation of near horizontal failure surface at residual conditions;

d) Suggests using (I think based on Fig 17 IC7)
   - Fully softened envelope along inclined failure surface
   - Residual " " horizontal"

Fig. 5. Typical shear stress-displacement curves and strength envelope for direct drained shear tests on Lower Oxford Clay specimens sheared parallel to the bedding.
Mobilized friction angles back-calculated from reactivated and first-time slope failures compared to the range from empirical information.

Figure by MIT OCW.
1.8 Basic Research on $\phi'$

Kenney (1967) Also Cuff (1977) ICMEG

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<thead>
<tr>
<th>Mineral</th>
<th>$% - 2m$</th>
<th>$k/e Salt$</th>
<th>$I_p$ (%)</th>
<th>$\phi'_r$ (tan$\phi'_r$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz</td>
<td>100</td>
<td>-</td>
<td>0</td>
<td>35 (0.70)</td>
</tr>
<tr>
<td>Alapulgit</td>
<td>74</td>
<td>- 345</td>
<td>240</td>
<td>29.6 (0.51)</td>
</tr>
<tr>
<td>Na Illite</td>
<td>100</td>
<td>0 51</td>
<td>18</td>
<td>16.2 (0.29)</td>
</tr>
<tr>
<td>Na. Mont.</td>
<td>100</td>
<td>0 1325</td>
<td>1270</td>
<td>16.0 (0.07)</td>
</tr>
</tbody>
</table>

Mixtures of massive & clay minerals

$$R_{\phi'_r} = \frac{\tan \phi'_r (Mixture) - \tan \phi'_r (Clay)}{\tan \phi'_r (Massive) - \tan \phi'_r (Clay)}$$

* See IC9

1.9 Empirical Correlations

- Voight (1973) Jot 23(2)
- Lupini et al (1981) Jot 31(2) + fundamental studies - See IC10
- Deere $\phi_r'$ in IC10
- Stanki Eid (1994) JCE, ASCE, 120(5) IC11

**CF < 252**

Low $I_p$: Relatively little particle orientation

High $I_p$: Significant particle orientation (can get highly polished surface)

**CF > 502**

Higher $\bar{\phi}$ than $\phi$

Lower $\bar{\phi}$ than $\phi$

Shepperton (1985)

Jot 35(1)
Kenney (1977) "Residual Strength of Mineral Mixtures"
9th ICSMFE Vol. 1 pp. 155-160

Results from repeated Direct shear at $\sigma_n' = 1$ kg/cm²

$$R_{\phi'} = \frac{\tan \phi'_{(Mixture)}}{\tan \phi'_{(Clay)}}$$

TABLE II. RESULTS OF RESIDUAL STRENGTH TESTS ON MIXTURES.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Mineral Content</th>
<th>Residual State $\sigma'' = 1.0$ kg/cm²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>% dry wt.</td>
<td>Wm (Res.) %</td>
</tr>
</tbody>
</table>

A. MIXTURES CONTAINING MONTMORILLONITE

| Montmorillonite - Na and Quartz | 0  | 50/50 | 40  | 50  | 200  | 92  | 0.00 |
| Montmorillonite - Na and Quartz | 0  | 25/75 | 25  | 75  | 96  | 53  | 0.11 |
| Montmorillonite - Na and Quartz | 0  | 10/90 | 10  | 90  | 44  | 60  | 0.42 |
| Montmorillonite - Na and Quartz | 0  | 5/95  | 5   | 95  | 59  | 54  | 0.61 |

B. MIXTURES CONTAINING KAOLINITE AND GRUNDITE

| Kaolinite and Quarz | 0  | 75/25 | 25  | 75  | 40  | 62  | 0.33 |
| Kaolinite and Quarz | 0  | 50/50 | 50  | 50  | 30  | 73  | 0.39 |
| Kaolinite and Quarz | 0  | 25/75 | 25  | 75  | 23  | 54  | 0.65 |
| Kaolinite and Quarz | 0  | 75/25 | 25  | 75  | 44  | 80  | 0.32 |
| Grundite and Quarz | 0  | 75/25 | 25  | 75  | 67  | 91  | 0.35 |
| Grundite and Quarz | 0  | 25/75 | 25  | 75  | 69  | 82  | 0.63 |

C. MIXTURES CONTAINING HYDROUS MICA

| Hydrox. mica I - Na and Quarz | 0  | 75/25 | 45  | 55  | 30  | 74  | 0.35 |
| Hydrox. mica I - Na and Quarz | 0  | 50/50 | 50  | 50  | 31  | 63  | 0.44 |
| Hydrox. mica I - K and Quarz | 0  | 75/25 | 45  | 55  | 33  | 66  | 0.46 |
| Hydrox. mica I - K and Quarz | 0  | 50/50 | 50  | 50  | 30  | 63  | 0.47 |
| Hydrox. mica II - Na and Quarz | 0  | 75/25 | 33  | 67  | 31  | 80  | 0.32 |
| Hydrox. mica II - Na and Quarz | 0  | 50/50 | 50  | 50  | 22  | 66  | 0.46 |
| Hydrox. mica III - Na and Quarz | 0  | 75/25 | 33  | 67  | 41  | 82  | 0.45 |
| Hydrox. mica III - Na and Quarz | 0  | 50/50 | 50  | 50  | 27  | 67  | 0.47 |

Fig. 4. Relative residual strength
DRAINED RESIDUAL STRENGTH OF COHESIVES SOILS

Residual Friction Angle, $\phi'_r$

Plasticity Index $I_p$ %

Residual Strength : Correlations with Plasticity Index

- Vaughan et al. (1978) $\sigma_n = 130 - 180$ kPa
- Bucher (1975) $\sigma_n = 72.5 - 269.5$ kPa
- Kanji (1974) $\sigma_n = 147$ kPa
- Seycek (1978) $\sigma_n = 300$ kPa
- Fleischer (1972)
- Voight (1973)

Figure by MIT OCW.
Effect of Clay Mineralogy on Drained Residual Failure Envelopes

Reduction in Secant Residual Friction Angle from Effective Normal Stresses of 50 kPa to 700 kPa

Relationship between Drained Residual Friction Angle and Liquid Limit

Figures by MIT OCW

Adapted from:
JGF, ASCE 120(5) (1994)
C2. HIGHLY STRUCTURED, SENSITIVE CLAYS (Quick Clays)

2.1 Background

- Almost flat slope (Norway)
- "Ming" = geometry of massive flow slide
- Picture 3rd floor
- Russian film

2.2 Norway Aas (1981) ICSEME

- Analysis of "flak" type slides: Treat as CO Case via USA

\[ \frac{\tau}{g_{vo}} \]

\[ \phi \]

Undrained condition: \( \kappa_{sh} > 0 \)

- CKD IDSS
- CK0 UDS

\[ \frac{\tau}{g_{vo}} = 0.18 \pm 0.035 \]

Lar CK0 UDS \( \frac{\tau}{g_{vo}} = 0.195 \pm 0.025 \)

2.3 Quebec Lefebre (1981) CGT p.920

- Treat as CO Case using equilibrium \( \tau \), but "large strain" values of \( \varepsilon \) & \( \phi \)

\[ \varepsilon \]

- Large strain \( (\varepsilon = 10\%) \) used to select \( \varepsilon \) & \( \phi \)

See ICSEME

CCL: Seems more empirical than 2.2, but applied to circular arc type failures
**B-2 CLAY**

- ENDS OF UNDRAINED STRESS PATHS \( \sigma_{vc} \) \( \rightarrow \) \( \sigma_{vo} \) \( \rightarrow \) \( 2 \sigma'_{p} \)
- POST PEAK STRENGTH FROM CIDCTESTS (Loading)
  - CIDCTESTS, Km 24.5
  - CIDCTESTS, Km 18.5

\( \phi' = 33^\circ \)

\( c' = 3.6 \text{ kPa} \)

\( \sigma'_{1} - \sigma'_{3} \)

**FIG. 5.5-9 TYPICAL EFFECTIVE STRENGTH ENVELOPES B-2**

*From SEBJ (not for publication)*
C. Effective Stress Envelopes for Conn. Valley Varved Clay


a) CDOS Parallel to Varves (Clay w/ $\sigma_p' = 3.5$ TSS)

--- Shear through "silt" layer
--- Shear in "clay" layer

Results $\rightarrow$ 3 envelopes depending on $\sigma_v'$ level

b) Summary of ESE Data  Bulk Ip $= 15-30\%$  Ladd & Foose (1977) FHWA

CU Shear Across Varves
Compression + Extension
SHANSEP Tests

$C/\sigma_v' = 0.012$  $\phi' = 30\%

CD Shear in Clay Varves

$C/\sigma_p' = 0.025$  $\phi' = 20\%$